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Significance of Foundation-Soil Separation in Dynamic Soil-Structure Interaction

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The dynamic response of flexible surface strip-foundations allowed to uplift is numerically obtained for externally applied forces of a transient time variation. The soil medium is represented by an isotropic, homogeneous and linear elastic half-space. The soil is treated by a time domain Boundary Element Method, while the flexible founadtion is treated by the Finite Element Method. In order to effectively simulate soil-foundation separation, thin-layer FEM interface elements are used at the contact area. The numerical procedure of determining the area of contact by solving the nonlinear equations of motion is based on the BEM and FEM appropriately combined through equilibrium and compatibility considerations. For various relative stiffnesses between the foundation and soil the system is subjected to a concentrated impulse force and/or moment acting on the surface foundation. It is observed that separation significantly affects the foundation response, and should be considered in the analysis for a range of relative stiffness between the foundation and the soil.

INTRODUCTION

Most soil-structure interaction problems are treated under the assumption of complete bond between the foundation and the soil [1-3]. However, for a given eccentricity and intensity of external dynamic forces, a foundation will partially separate from the underlying soil, as tension is incompatible with the constitutive laws of soils. Recently, some attention has been directed towards the study of the effects that partial foundation-soil separation may cause on the structure response [4-6]. These studies have been initiated from observations during strong ground motions, actual performance of structures during earthquakes and laboratory tests [7-10]. Both analytical studies and numerical investigations demonstrated that uplift may have controversial effects on structural behavior. Factors such as slenderness ratio, foundation to superstructure mass ratio, eigen properties of the structure, type and duration of the exciting disturbance may have either benevolent or malevolent effects on the structure response.

The methods of non-linear analysis usually employed to obtain the structure response can be classified into three categories: a. Employment of discrete systems idealizing the foundation in a small number, usually two, of elasto-plastic springs and ignoring both the radiation damping and the coupling between the soil-foundation contact stresses at the time of separation [11]; b. Simulation of the soil behavior by either a damped Winkler foundation or a foundation supported on two elastic spring-dampers attached at the ends [4,5,12]; c. Employment of a finite difference [8] or finite element method (FEM) of analysis [13,14] to model soil media leading to a large system of equations. Recently, Wolf et al. [15,16] determined the response of a typical nuclear-reactor building modeled by a single degree of freedom in the vertical direction supported by a rigid circular foundation subjected to vertically incident seismic waves. Their formulation is based on a time domain indirect boundary element formulation (BEM) employing an inverse Fourier transform on the level of the individual boundary elements.

In this paper, the dynamic response of massless flexible surface strip-foundations allowed to uplift is numerically obtained for externally applied forces of a transient time variation. The soil medium is represented by a homogeneous and linear elastic half-space. The soil is treated by the BEM, while the foundation and the interface are treated with the aid of FEM. The numerical procedure of determining the area of contact by solving the nonlinear equations of motion is based on the BEM and FEM appropriately combined through equilibrium and compatibilty considerations. Thus, the formulation does not require the adoption of frequency independent compliances needed for the solution of nonlinear dynamic soil-structure interaction problems. The primary contributions of this work are the development of a methodology that allows a rigorous treatment of the separation effects on soil-structure interaction problems as well as a thorough investigation of the influence of uplift on the response of flexible surface strip-foundations.

METHODOLOGY

Consider the soil-structure system of figure 1, which is allowed to oscillate with unilateral contact. The foundation and the interface are treated with the aid of FEM, while the soil is treated by the BEM. The two domains are appropriately combined through equilibrium and compatibility considerations at the soil-foundation interface. The interface is modeled with thin-layer elements of negligible influence on the system response. The treatment of the thin-layer elements simulating the interface behavior is discussed in the next section. In the following, the treatment of the soil and the foundation is briefly discussed.

Under the assumptions of zero initial conditions and zero body forces, the BEM formulation is developed through a numerical treatment of the integral equation governing the soil motion at the soil-foundation interface having the form [17,18]

$$\tfrac{1}{2}u_\beta(\xi,t)=\int_s\{v_{\alpha\beta}[x,t;\xi/t_{(\tilde n)\alpha}(x,t)] - -\sigma_{(\tilde n)\alpha\beta}[x,t;\xi/u_\alpha(x,t)]\}ds(x), \qquad (1)$$

where s denotes the soil-foundation interface as well as a portion of the free surface around it, and the tensors $v_{\alpha\beta}$ and $\sigma_{(\tilde n)\alpha\beta}$ represent the fundamental solution pair of the infinite space under conditions of plane strain.

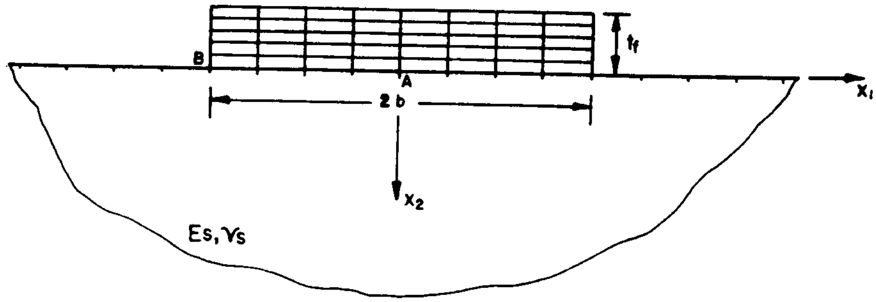


Fig. 1 Soil-structure modelling

The numerical treatment of the boundary integral equation (1) involves both time and spatial discretization. Thus, the time variation of $t_{(\tilde n)}(x,t)$ is approximated as a sequence of rectangular impulses of equal duration Δt. The soil-foundation interface, as well as a part of the surrounding free soil surface are dicretized into Q elements of equal length L. The foundation response at time $t=N\Delta t$ due to a sequence of impulses initiating at time $m\Delta t$ can be determined from

$$\tfrac{1}{2}u^{Np}=\sum_{q=1}^{Q}\sum_{n=m}^{N}\Big([\int_{\Delta s}G^{1q}ds]\{t^{N-1+1}\} - -[\int_{\Delta s}F^{1q}ds]\{u^{N-1+1}\}\Big), \qquad (2)$$

where G^{1q} and F^{1q} are the discretized kernel functions

$$v_{\alpha\beta}[x,t;\xi/t_{(\tilde n)\alpha}(x,t)] \quad \text{and} \quad \sigma_{(\tilde n)\alpha\beta}[x,t;\xi/u_\alpha(x,t)]$$

respectively, $n=1,2,\ldots,N$, $1=N+n-1$, $q=1,2,\ldots,Q$, and $p=1,2,\ldots,Q$.

The other component of the system, the flexible foundation is analyzed through standard finite element procedures. The discretization is carried out using four node rectangular isoparametric plane-strain finite elements. The dynamic equation of the foundation motion is given by

$$[M_f]\{\ddot q_t\}+[C_t^f]\{\dot q_t\}+[K_t^f]\{q_t\}=\{R_t\}-\{P_t\} \qquad (3)$$

where $[M_f]$ is the mass matrix, $[C_t^f]$ and $[K_t^f]$ are the time dependent damping and stiffness matrices respectively, the vectors $\{\ddot{q}_t\}$, $\{\dot{q}_t\}$ and $\{q_t\}$ are the nodal acceleration, velocity and displacement vectors, respectively, the vectors $\{R_t\}$ and $\{P_t\}$ are the nodal external forces and nodal forces associated with the contact stresses. The matrices $[C_t^I]$ and $[k_t^I]$ are time dependent because they contain the terms pertaining to thin-layer interface elements. The propertiese of the thin-layer elements are dependent on the contact area which is a function of time.

Equation (2) relates the average vertical displacements at the center of each element to the contact stresses developed over the elements of the soil-foundation interface. Equation (3), relates the vertical nodal displacement to the nodal forces associated with the contact stresses developed at the ends of the FEM elements at the interface. In order to introduce compatibility between the deflection of the foundation and the soil motion at the interface, the average displacement over an element q is approximated by the mean value of the nodal displacements at the ends of the element q. Similarly, compatibility of forces can be established if each contact force P_t applied at a node i is approximated by the mean value of the two resultant forces R_t associated with the contact stresses that develop over two successive elements joined at the common node i. Thus, for the whole interface region the compatibility relationships can be expressed as

$$\{q_t\}=[T]\{u_t\}$$
and
$$\{P_t\}=[T]^T\{R_t\} \tag{4}$$

where the entries of matrix [T] are either 0 or 1/2. The order of matrix [T] is Qx(Q+1).

Combination of equations (2), (3) and (4) results in a system of nonlinear equations of motion

$$[M]\{\ddot{q}_t\}+[C_t]\{\dot{q}_t\}+[K_t]\{q_t\}=\{F_t\}-\{P_t\} \tag{5}$$

All quantities in equation (5) are known at a given time. Equation (5) is solved iteratively to satisfy the time dependent boundary conditions at the soil-foundation interface. The contact area at the beginning of each time step is known from the iterative solution of the previous time step. Thus equation (5) at time $t+\Delta t$, where Δt is a small time increment, can be written as

$$[M]\{\Delta\ddot{q}_t^{\,i+1}\}+[C_t]\{\Delta\dot{q}_t^{\,i+1}\} + \\ + [k_t]\{\Delta q_t^{\,i+1}\}=\{\Delta R_t\}+\{R_t^{\,n,i}\} \tag{6}$$

where $\{q_{t+\Delta t}\}=\{q_t\}+\{\Delta q_t\}$,

$[K_{t+\Delta t}]=[K_t]+[\Delta K_t]$, etc.

and $\{\Delta R_t^{\,n,i}\}$ is the unknown nonlinear load vector corresponding to the time increment Δt to be determined by iteration and i is the number of iteration within the same time step. The vector $\{\Delta R_t^{\,n,i}\}$

is given by

$$\{\Delta R_t^{n,i}\}=-[\Delta C_t^{\ i}]\{\dot{q}_{t+\Delta t}^{\ i}\}-[\Delta K_t^{\ i}]\{q_{t+\Delta t}^{\ i}\} \tag{7}$$

An unconditionally stable scheme of direct integration based on Wilson θ method is used in the time domain. At the desired time $t+\Delta t$ the accelerations, velocities and displacements are given by the linear acceleration assumptions:

$$\{\ddot{q}_{t+\Delta t}\}=(1-1/\theta)\ddot{q}_t+(1/\theta)\ddot{q}_{t+\tau} \tag{8}$$

$$\{\dot{q}_{t+\Delta t}\}=\dot{q}_t+(\Delta t/2)(\ddot{q}_t+\ddot{q}_{t+\Delta t}) \tag{9}$$

$$\{q_{t+\Delta t}\}=q_t+\Delta t\dot{q}_t+(\Delta t^2/6)(\ddot{q}_{t+\Delta t}+2\ddot{q}_t) \tag{10}$$

where τ is given by $\tau=\theta\Delta t$. When $\theta=1.0$ the algorithm reduces to the standard linear acceleration method. A stability analysis reported by Wilson, Farhoomand and Bathe [19] shows that the scheme is unconditionally stable provided $\theta\geq1.37$.

THIN-LAYER INTERFACE ELEMENTS

In order to simulate unilateral contact at the soil foundation interface, the interface is modeled with the aid of FEM thin-layer elements of negligible influence on the system response. The interface element can undergo four basic modes of deformation. (1) Stick or no-slip, (2) slip or sliding, (3) separation or debonding; and (4) rebonding. An interface element is in stick mode when there is no relative motion between the adjoining bodies. If a relative movement takes place while maintaining the contact between the adjoining bodies, the slip or sliding is said to occur. Separation or debonding takes place when the bodies open up due to contraints of unilateral contact. If the interface element in separation mode returns to stick mode in subsequent loading, rebonding takes place.

The interface element described above has been successfully used for solution of a number of static as well as dynamic two-demensional problems where all domains are discretized with the aid of FEM [14,20]. In this study, the equations of the interface elements are derived separately and then added to those of the foundation prior to establishing the compatibility and equilibrium criteria with the soil BEM modeling described in the previous section.

The primary reason for resorting to interface elements at the interface is to facilitate the computation of the contact area prior to each time step. The interface element when in stick mode is essentially treated like any other plane strain element with the soil elastic modulus, E_s, and Poisson ratio, v_s. In the present study, the concept of sliding is not addressed. In debonding mode of a given interface element, the elastic modulus is assigned a value of zero. This in essence creates a void element with no stiffness. Within a given cycle of iterations in a time step, if rebonding is detected through interpenetration, the forces associated with the contact stresses are applied to the penetrating node. Thus stick mode of a previously defined void element can be stimulated without modification to the global stiffness matrix $[K_t^f]$. The interface element thickness plays an important role in the convergence of the solution as reported by several researchers [14,20]. In this study the ratio between the thickness of the interface element, and the thickness of its neighbouring FEM element is taken as 0.01.

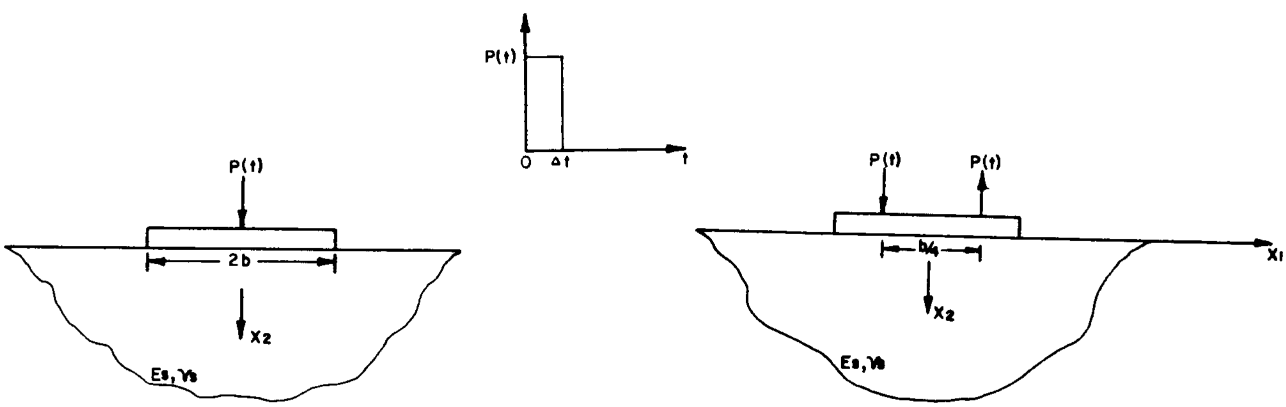
NUMERICAL EXAMPLES

The combined time domain BEM-FEM technique described above is employed here to determine the dynamic response of a flexible massless strip-footing subjected to externally applied loads. The dynamic behavior of undamped flexible footing depends on the special distribution of the externally applied forces and by the material properties of the elastic footing. Therefore, the footing and the supporting elastic medium are analyzed in this work for three sets of elastic contstants and two types of exernal forces (figure 2). The parameter characterizing the flexibility of the soil-foundation system is the relative stiffness defined by

$$K_r = D_f \cdot D_s \qquad (11)$$

where $D_f = E_f t_f^3/(1-v_f^2)$ and $D_s = 2(1-v_s)/(E_s b^3)$ (12)

and where the subscript f and s denotes the footing and the soil, respectively, E and v represent modulus of elasticity and poisson's ratio, respectively, and t_f is the thickness of the footing.



(a) Central concentrated load (b) Force-couple load

Fig. 2 Loadings considered

The soil is discretised into 16 BEM elements and the foundation is discretised into 40 FEM elements as shown in figure 1. Figure 2 shows the two types of external loadings considered, the point force and a moment applied as a force couple of two equal, opposite point forces. The duration of both impulse forces is $\Delta t = 0.16\times10^{-4}$ sec, and the relative stiffness considered are $K_r = 0.3$, $K_r = 3.0$ and $K_r = 30.0$. The response of the center point A, and edge point B of figure 1 are plotted in figure 3 through 5. All responses are compared with the corresponding solution of the complete bond case, i.e. uplift not permitted.

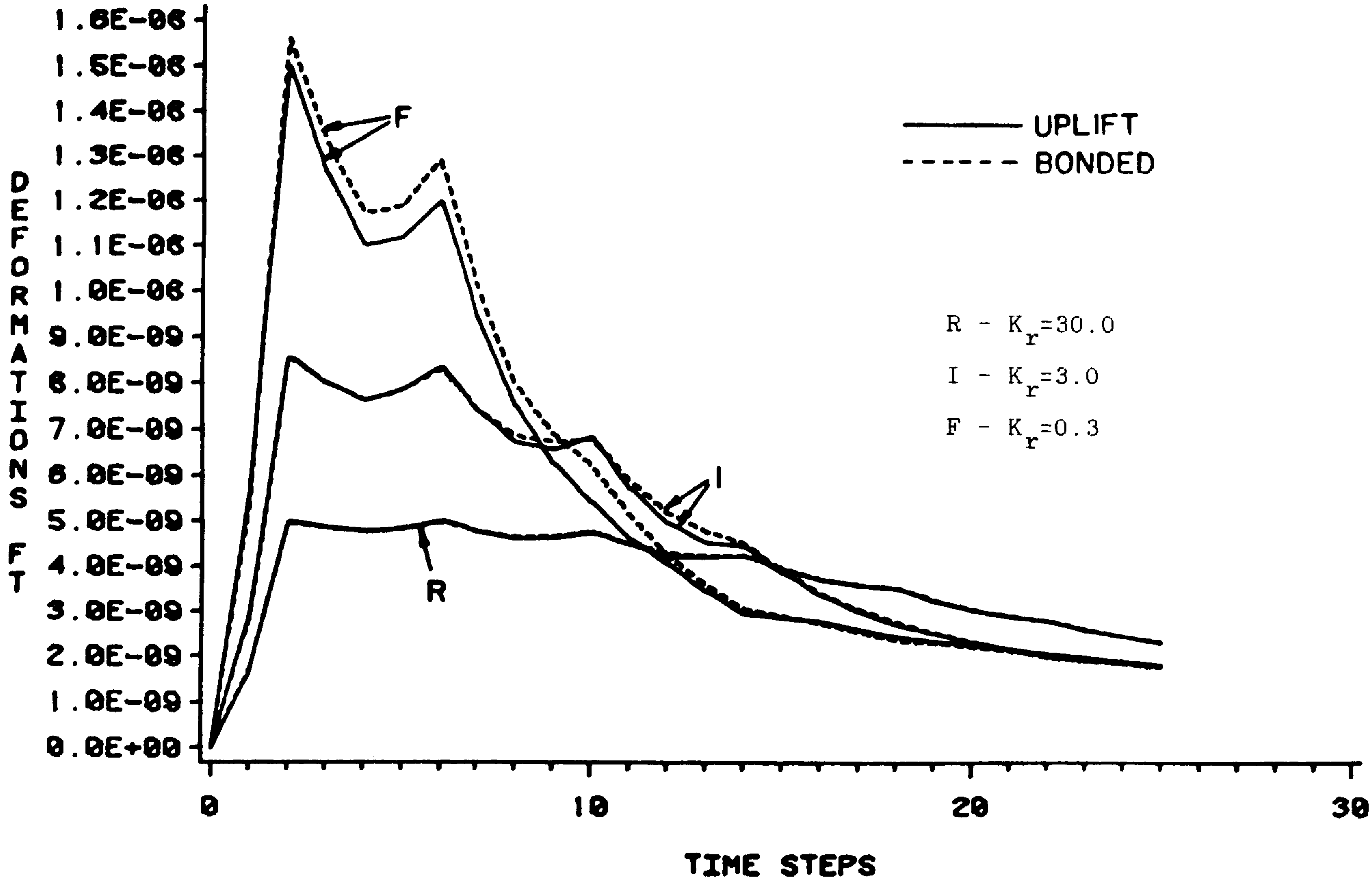


Fig. 3 Midpoint Deflection due to central concentrated load

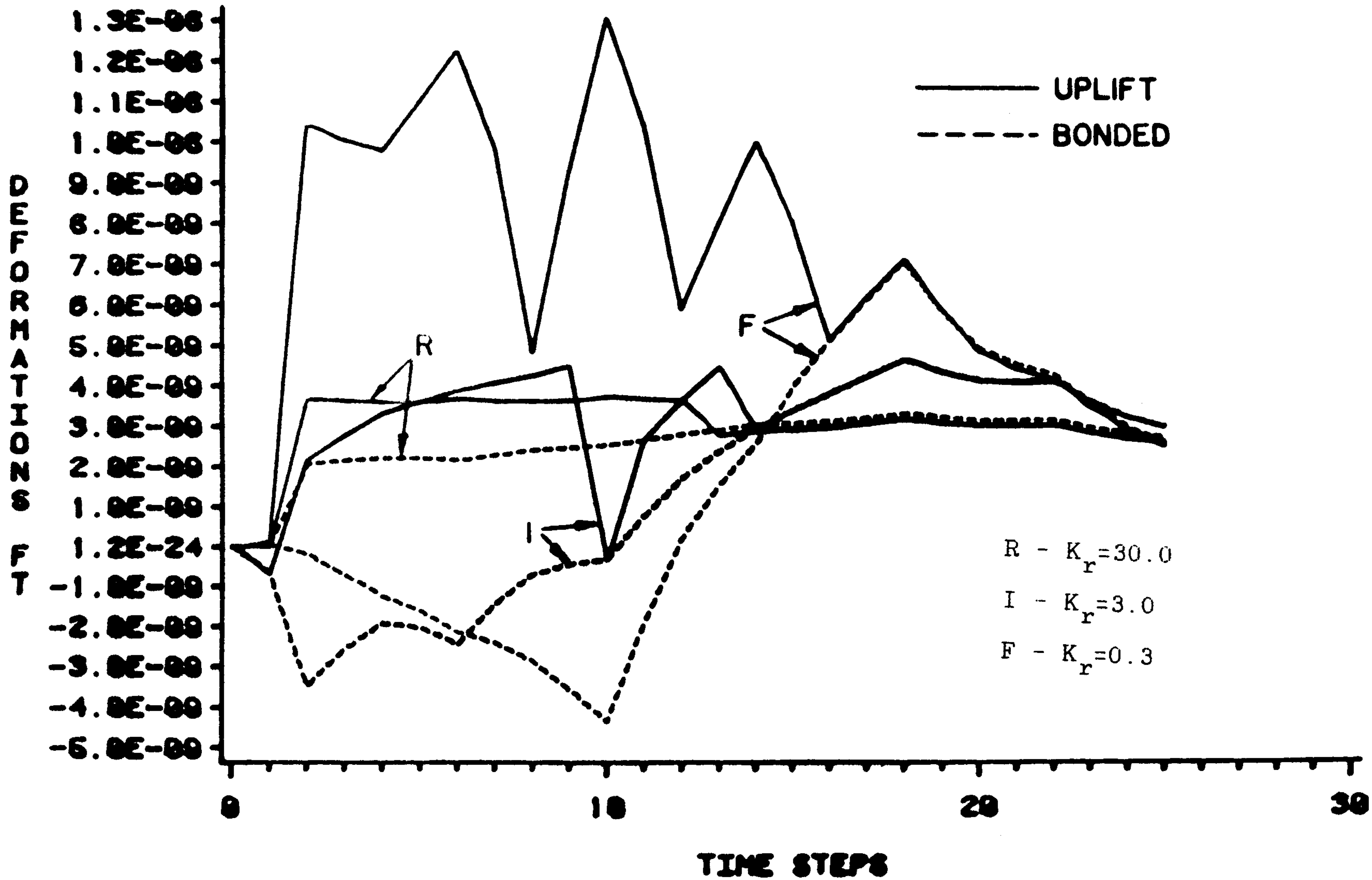


Fig. 4 Edgepoint deflection due to central concentrated load

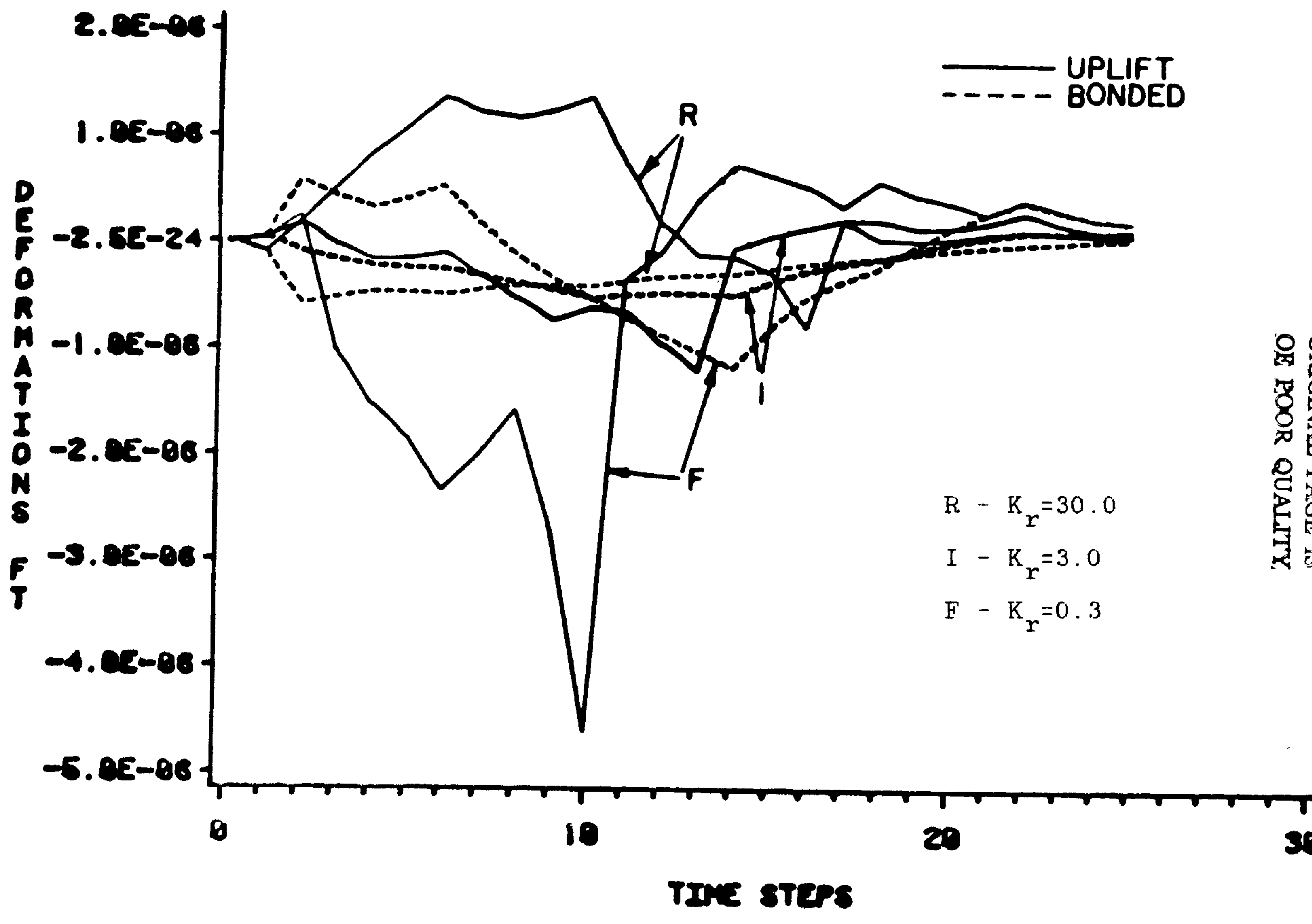


Fig. 5 Edgepoint deflection due to force-couple load

The responses at the midpoint A and the edge point B of figure 1 due to a vertcal concentrated rectangular impulse load at midpoint A are plotted for unilateral and bilateral contact for various relative stiffnesses (K_r=0.3, 3.0, 30.0). As seen from figure 3, the response at the center for the unilateral contact is higher then the corresponding bilateral contact case. Figure 4 shows that the deformations at the edge point to be significantly higher for the case of unilateral contact then that for the bilateral case. The deformations are in the opposite sense because the foundation is not held back as tension is incompatible with the assumed constitutive laws of the soil (unilateral contact). At both, the center point and edge point locations the differences between the unilateral and bilateral contact conditions decreases with increasing foundation stiffness.

In the case of force couple loading, the softer foundation K_r=0.3 and stiffer foundation K_r=30.0 undergo higher deformation differences than the intermediate stiffness K_r=3.0 as shown in figure 5. The deformations become identical with the passage of time as seen for the concentrated load case.

CONCLUSIONS

It can be concluded that intermediate relative stiffness leads to moderate deformations when uplift is permitted. Very flexible footings produces higher deformations in unilateral contact compared to bilateral contact, and thus should be considered in their design. Unilateral contact does not significantly increase deformations for stiff footings subjected to concentrated central loading. However, relatively large deformation differences occur when the loading is eccentric necessitating consideration of uplift in their design.

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